THE PORT BOUVARD BRIDGE

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ABSTRACT: The Port Bouvard Bridge over the Dawesville Channel 100 km south of Perth was built by Thiess Contractors and opened to traffic in September 1993, two months ahead of schedule. The bridge was incrementally launched, and the design contained many features which facilitated meeting the weekly cycle which was achieved for all launches. It has a number of unusual design aspects which combined to produce an easily constructible and maintainable bridge. The most striking of these is the very large open cross-section, which utilises a pair of 3.1 metre deep I beams to support a 19 metre wide deck.

With a length of 360 metres, this is the largest bridge designed in Western Australia since the introduction of the Austroads Bridge Code, which proved to have some unexpected influences on the design. The substructure columns are up to 19 metres high and the pilecaps were specially designed to resist vessel impact. The bridge also has special features for services access, maintenance, and pedestrian facilities.

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1. INTRODUCTION:

1.1 General

A major environmental construction project is almost completed in Western Australia at Dawesville, near Mandurah, 100 kilometres south of Perth. The work is being carried out by Thiess Contractors Pty Ltd for the Department of Marine and Harbours Western Australia. A channel with depth varying from 4.5 to 7 metres and up to 200 metres wide has been cut across a 1 kilometre wide isthmus separating the Indian Ocean and the Peel/Harvey Estuary, which is fed from inland by the Harvey River (see Figure 1). The channel will provide tidal flushing to increase the salinity of the inlet and counter the effects of fertilisers which have entered the inlet system, causing periodic algal blooms, and creating an undesirable marine environment. Where the channel cuts across the main road heading south towards Bunbury a new bridge has been constructed, and for this part of the project the commission was jointly issued by the Department of Marine and Harbours and Main Roads Western Australia. As part of its total contract, Thiess arranged for the design of this bridge and chose Bruechle Gilchrist and Evans Pty Ltd as their consultant. The Port Bouvard Bridge forms part of a growing trend for types of contract in which the contractor decides the form of the bridge, which may be by "design and construct" bidding or some form of negotiated contract, as in this instance. Given the choice, it is becoming increasingly clear that incremental launching is a method often favoured by the contractors, and this was the construction technique preferred by Thiess.

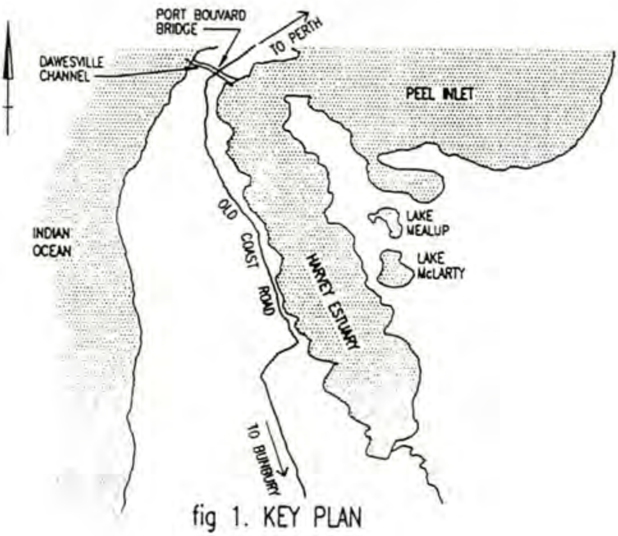
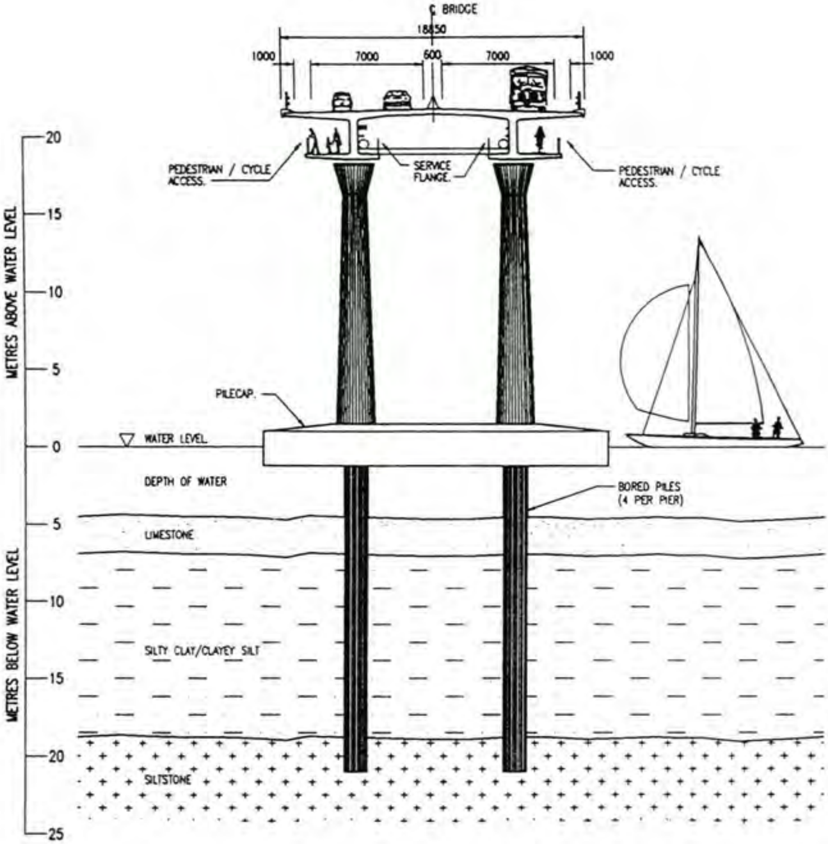
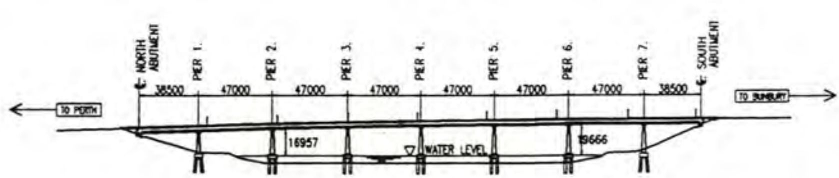


fig 1. KEY PLAN



SECTION



ELEVATION

fig 2. OVERVIEW OF PORT BOUVARD BRIDGE

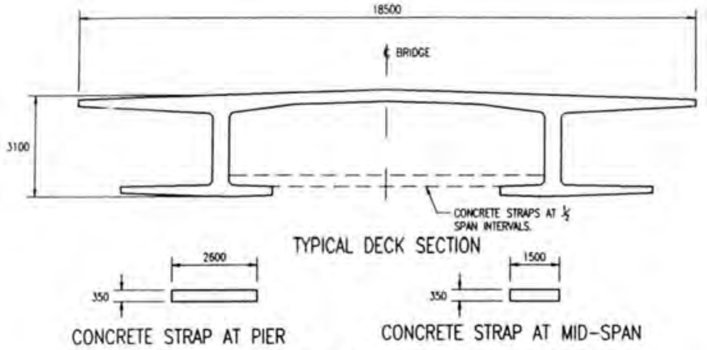


fig 3. CROSS SECTION

1.2 The Bridge Superstructure

The bridge is 359 metres long, 19 metres wide, 3.1 metres deep, and comprises 6 internal spans of 47 metres, and two 38.5 metre end spans. A overview of the structure is provided in Figure 2. The most striking unusual feature of the superstructure is the very large open section, which utilises a pair of full depth I beams to support the deck (see Figure 3), instead of the conventional box girder. The bridge is designed to the requirements of Main Roads Western Australia for special heavy vehicles specific to this route, including the largest vehicle specified in the new Austroads Bridge Code, the HLP400. It has two walkways at the level of the lower flange, so that pedestrian and cycleway traffic is separated from road traffic. Another unusual feature of the superstructure is the lack of web supporting diaphragms, which allows a multiple array of services to be easily carried across on the internal flanges. Although it is not anticipated that much maintenance will be required, access for inspection and maintenance is excellent to all points of the structure.

1.3 The Bridge Substructure

The abutments are founded on spread footings and internal piers on piles. At both abutments there is a pedestrians' viewing area to take advantage of this spectacular location. Each internal pier comprises a pair of elongated truncated cone columns, with a short inverted truncated cone capital. Extensive use was made of the scale models and three dimensional computer imaging prepared by Main Roads Western Australia to assist in choosing an architecturally pleasing solution, especially for the piers, which was at the same time cost effective to build. There is a 19 metre clearance for yacht masts in the main navigation span, as required by the Department of Marine and Harbours, who also required the pile caps to resist the impact of a 300 tonne vessel travelling at 8 knots in the main navigation span. Their shape is especially designed to help achieve this. The two internal piers on land are each

supported on twenty 550 mm diameter Frankipiles, about 13 metres long, while the internal piers in the channel are each supported on four 1.5 metre diameter bored piles founded in siltstone approximately 20 metres below sea level. This design was the outcome of a performance specification for the piles, for which the successful tenderer was Frankipile. For maximum durability performance the concrete in both the piers and pile caps uses a high blend slag mix with a low water cement ratio and incorporates silica fume.

2. THE INCREMENTAL LAUNCHING CONSTRUCTION PROCESS:

The bridge was constructed by the incremental launching technique, introduced into Australia by Bruechle Gilchrist and Evans in 1984, and many features of the way it is designed are directed to streamline this type of construction process. To use this process economically, it is essential to maintain a weekly cycle in constructing the superstructure, and the order and timing of all processes must dovetail to achieve that end. Incremental launching had a number of advantages at this location, some of which are as follows:

a. The substructure could be built in parallel with the superstructure, as long as the piers were available at least one span ahead. This enabled a considerable compression in project completion time, and allowed great flexibility in deployment of the labour force.

b. Nearly all construction on the superstructure was carried out in the casting bed area, making quality control a much more local and therefore simpler task. The types of forms used produce very high quality finishes, akin to very good precast work, and because the concrete is required to reach 30 MPa in three days that type of mix design ensures very good durability even in this relatively aggressive environment.

c. The formwork was high quality and precise, and although in itself relatively expensive, becomes very cost effective when used over the 23 launches. In this project the side and deck forms were steel, while the casting bed was precision cast concrete separated from the cast segments with a special layer of sliding foil.

d. The incremental launch method tends to use a relatively small constant labour force.

e. Services could begin to be installed before construction of the superstructure was complete, again compressing construction time, minimising handling of service pipes which were launched along with the bridge, and allowing efficient use of the labour force.

The construction method was a complete success. A cycle was completed every week except for the Christmas-New Year and Easter breaks. The bridge was completed two months ahead of schedule, and workmanship and finish is of outstanding quality throughout the structure.

3. THE SUPERSTRUCTURE:

3.1 General

The most unusual feature of this bridge is the cross-sectional shape, which is really a large pair of I-beams (see Figure 3). The other very unusual feature is the lack of full height diaphragms. Apart from the deck slab, the only transverse connection is a series of concrete 'straps' at half span intervals, which control the stresses caused by transverse bending in the lower flanges (see Figure 3). This configuration was initially chosen to assist the construction process in several ways as described immediately below:

a. It enables the launching operation to proceed with minimal manoeuvring of formwork. Essentially the bridge was launched straight out of the forms. This was made even more simple by the omission of full height transverse diaphragms, and the maintenance of a constant cross-sectional width, except where web thicknesses varied in the end three metres of the bridge.

b. The twin beam elements are much less sensitive to launching tolerances than a box girder would have been. This is especially true for transverse tolerances.

c. Given that an initial Main Roads requirement was that lower level footways be provided, the twin I-beam solution used a reduced amount of concrete in cross-section compared with a box girder. A corollary benefit was that it enabled the use of vertical webs without a penalty in increased lower flange concrete.

3.2 Details for Efficient Construction

Some special features of the superstructure design and some techniques adopted by the contractor which assisted in meeting the construction schedule are as follows:

a. Great care was taken to ensure that there were generally only two planes of reinforcement in the upper and lower flanges, and especially the web. This enabled ease of fabrication of reinforcement, and ease of installation of prestressed cable ducts. It also ensured good access for vibration of the concrete, and well compacted concrete has been universally achieved throughout the structure.

b. Extensive use was made of prefabricated reinforcement cages, prepared on site adjacent to the casting bed. The lower flange steel and web steel was mostly prefabricated for use in the first stage cast, and upper flange (deck) steel placed in situ for the second stage cast. At what would otherwise have been very congested locations for spalling reinforcement at the end of each launch, welded, prefabricated reinforcement cages were prepared.

c. Extensive use was made of Lenton splices (full tension mechanical reinforcement connectors) at construction joints where concrete had to be added after the section emerges from the casting bed or after completion of all launches. These include details associated with continuity (as opposed to launching) prestress terminations, installation of the concrete 'straps', and upward extensions of the abutments.

3.3 Benefits of the Superstructure in Service

Although the cross-sectional shape was chosen initially as an aesthetic solution which would be easy to construct, Main Roads Western Australia were involved at an early stage, and it was perceived that there was a number of benefits for them as the final owners. These were:

a. There is easy maintenance access to services. These are located in an architecturally unobtrusive place above the internal flanges in between the webs. They are very accessible via a secure walkway along each side, and the open section also allows access from below. All manner of services, including gas (which cannot be carried through a box girder), can be included in this space which is fairly well protected from the corrosive environment.

b. All points on the superstructure can be easily inspected without much special equipment.

c. The vertical webs allow for an efficient attractive clear space for pedestrians. Sloping webs on lower slab level footways under box girder bridges result in an area which cannot be used because of the slope overhang. The available open space with the vertical webs is further enhanced because the gas service lines which cannot normally be included inside a box girder do not have to be placed in the footway area.

4. THE SUBSTRUCTURE:

4.1 Pier Columns

Some special features of the pier column design and some techniques adopted by the contractor which have assisted in meeting the construction schedule are as follows:

a. The column shape (see Figure 4) was carefully chosen so that the same forms could be used for all columns, although they are of varying heights.

b. The column capitals have sufficient area to allow installation of jacks for removal of the temporary bearings after launching, or maintenance of permanent bearings. There is also sufficient space for storage of the permanent bearings during construction ready for final installation.

c. Welded prefabricated steel was used at the top of pier column capitals where the requirements for spalling reinforcement for the temporary bearing removal caused congestion.

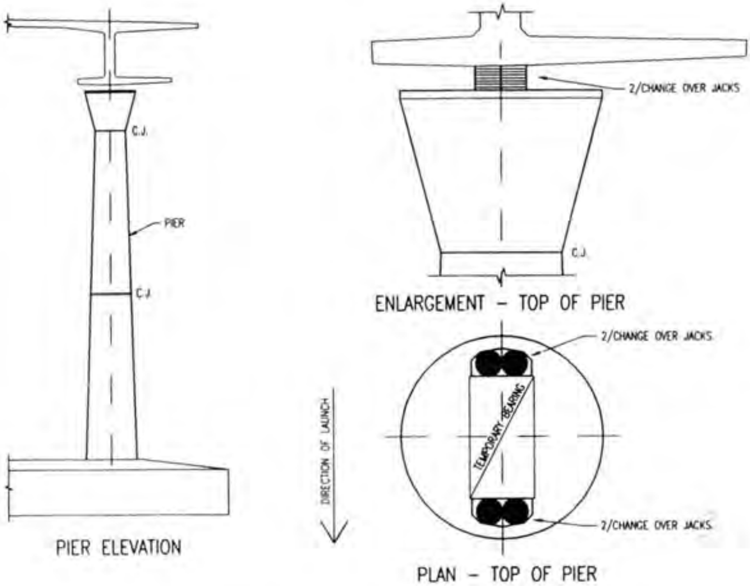
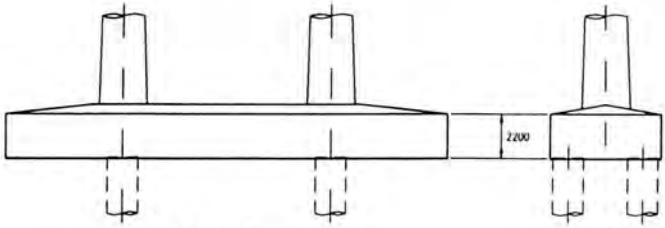


fig 4. PIER COLUMN

4.2 Pilecap and Piles

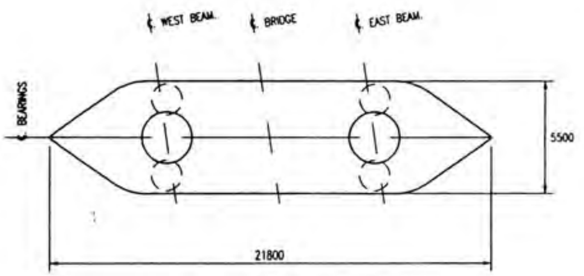
The Department of Marine and Harbours required a 19 metre clearance for yacht masts in the main navigation span. They also required a capacity to withstand the impact of a 300 tonne vessel travelling at eight knots in the main navigation channel. This decision was based on the largest ferry which might be used to carry passengers to Rottnest Island, and with the velocities of tidal flow in the channel, it was considered that eight knots would be required for steerage. Note that there are no applicable standards or codes for vessel collision loads, which left the Department of Marine and Harbours with the responsibility for making this determination.

In the unlikely event that permission is sought to use even larger vessels, a condition of the permit could be the installation of a fendering system. A fendering system was investigated for the present loading, but both Main Roads Western Australia and the Department of Marine and Harbours preferred another option, which is more architecturally acceptable and has no maintenance requirements. From Thiess's point of view it was also a lower cost option.

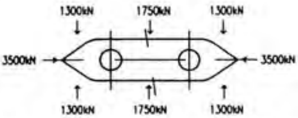


ELEVATION

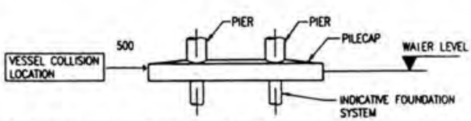
END ELEVATION



PILECAP PLAN



PLAN OF VESSEL COLLISION FORCES ON PILECAPS



ELEVATION OF VESSEL COLLISION FORCES ON PILECAPS

fig 5. PILECAP

The chosen solution is to design the pilecap and supporting piles to resist the vessel impact. In the end this choice made little difference to the pile design which was enveloped by other load cases. The principal effect was on the pilecap shape which is cast to have quite a distinct 'sharp' end (see Figure 5). This ensures that vessels travelling transverse to the bridge alignment strike the pilecap with a glancing blow, which considerably reduces the design forces. The method used is based on the principle of Conservation of Energy and the forces were generally derived in accordance with US DoTFHA (1990)[1]. The latter report recommends that 100% of the impact forces be used for forces transverse to the bridge alignment, and 50% for forces parallel to the bridge alignment i.e. drifting vessels. In this relatively narrow channel, the 50% seems somewhat conservative but was adopted nonetheless without particularly penalising the design. Pilecaps outside the main navigation channel were assessed for forces 50% of those in the main navigation channel, which was considered a reasonable allowance for a vessel drifting at about 4 knots with some windage. The latter case was not important in the overall design envelope.

5. ASPECTS OF DESIGN AND THE INFLUENCE OF THE AUSTROADS BRIDGE CODE:

5.1 Important Load Cases

5.1.1 Launching Tolerances

Bruechle Gilchrist and Evans have adopted the practice of designing for total vertical launching differential tolerance ranges of 5 mm longitudinally and 2 mm transversely. These are more generous tolerances than are often specified, but there seems no reason not to allow them, provided that it can be demonstrated that they do not cause excessive stresses. It has previously been the practice, especially as advised by German consultants, to specify very tight tolerance ranges such as +/- 1 mm longitudinally, but experience shows that these values present significant surveying verification problems. It is difficult to believe in practice that such tight tolerances could be demonstrated to have been met. In the analysis for launching for both ultimate and serviceability limit states the bridge is settled through the full range of possible longitudinal vertical tolerances, and temperature gradient effects are also allowed for. Although the larger tolerances will result in some extra concentric prestressing steel, it then becomes possible to assure that all design criteria have been met in the construction process.

5.1.2 HLP 400

The Port Bouvard Bridge is on a special heavy load route, and the entire suite of vehicles for which the route had previously allowed was included in the design envelope. In addition the Austroads (1992)[2] HLP 400 vehicle was included. It was by far the dominant load for longitudinal positive moment, longitudinal vertical shear and many aspects of the transverse design. This theoretical vehicle also dominated bearing loads and vertical loads to columns, while even the maximum special vehicle configurations for which the route might actually be used only produced design envelopes similar to Austroads (1992)[2] T44 and L44 envelopes. The authors have experience in other bridge design cases of the strong influence of the HLP 400, and even the HLP 320, and a parametric study by Wyche (1992)[3] has shown that they are

likely to dominate most designs where the spans exceed 30 metres, or even less for a smaller number of lanes. It is particularly disadvantageous that they so dominate the positive moment envelope, because it then becomes very difficult to adjust this envelope by using prestress secondary bending moments which tend to be positive anyway. In short the HLP 400, and possibly even the HLP 320 produce effects that are out of harmony with the rest of the design vehicles, even on a heavy load route. It seems hard to justify such heavy short vehicles being a standard design requirement, when they would be controlled by a permit issue system anyway. It is also clear that vehicles of this type would rarely, if ever, have obtained a permit in the past, particularly on a load route containing any bridges with spans greater than 30 metres. These new heavy load criteria specified in Austroads (1992)[2] are likely to have a significant cost impact on many future bridges, and theoretically make many existing bridges functionally obsolete.

5.2 Structural Behaviour of the Superstructure

Previously, bridges of the proportions of the Port Bouvard Bridge have been concrete box girders. For smaller, narrower bridges, twin T-beams would be used, and in the latter case the torsional stiffness of the T-beams would have a significant influence on the transverse slab flexure. However for a full size single cell box girder, although the torsional properties of the box may have some global influence on the individual web vertical deflections and therefore transverse slab flexure, the more dominant global effect is simply the vertical flexure of each web. This will be even more true if there are no web-restraining diaphragms, allowing transverse flexural distortion of the box which greatly reduces the torsional shear flow. In that sense the twin I-beam solution is much more like a single cell box girder than a large twin T-beam.

Transverse slab flexure is determined by portal frame action of the top slab and the webs, with the transverse position of the bottom of the frame (bottom of the web) controlled by the bottom slab in the box girder, or by the lower flanges in the twin I-beam. Initially it was intended to have lower flange horizontal restraint only at each pier, but the magnitude of the in plane lower flange moments and shears proved to be large enough to require structural connection at the mid span points as well. In the actual construction operation, installation of these 'concrete straps' proved to be relatively simple, and it may be sensible in future cases to use more of them to reduce lower flange steel.

5.3 Design of the Webs

For simplicity of bridge construction and simple installation of easily accessible services, it was decided to keep the webs a constant thickness, and avoid the use of web restraining diaphragms. For global and transverse flexural effects this arrangement behaves structurally as described above, but a penalty in reinforcing steel must be paid because of increased web moments, especially around the points of web restraint caused by the 'concrete straps'. A basic array of web steel is first determined by launching shear, and because the webs contain open continuity ducts during launching and are only 500 mm thick anyway, this necessitated a moderate quantity of steel along the length of the structure. However much of this later proved useful for web flexure. Less steel could have been used if the webs were made thicker, but experience shows that for shear design to Austroads (1992)[2], increasing shear

capacity by thickening webs is far less effective and economic than increasing vertical steel.

The very dominant load case for web steel eventually proved to be the HLP 400, but considerable analytical effort had to be expended to obtain envelopes of design cases with correctly associated simultaneous effects to determine a practical and economic arrangement of steel in the webs. The web bending which can occur in conjunction with maximum longitudinal beam shear will itself vary with transverse vehicle location, and the correct coincidence of these load cases must be assessed along the length of the structure to determine the best web steel layout. In addition to web moments and the longitudinal shear envelope, the usual other factors which also influence the quantity and longitudinal distribution of web steel must also be taken into account. These include vertical component of prestress, whether flexure shear (with maximum shear and associated moment, and maximum moment and associated shear) or web shear (principal tensile stress) governs, and in the latter case the internal compatibility stresses where relevant temperature cases are involved.

5.4 Longitudinal (Interface) Shear Steel

Longitudinal shear steel was designed in accordance with Austroads (1992)[2], and the most notable feature of the design was the quantity of steel required, compared with the previous experience of the authors. The code procedure is based on the steel or concrete areas outside the web flange interfaces which contribute to moment capacity. The contribution to bending moment strength of the concentric prestress which is placed outside the web faces is therefore responsible for large quantities of longitudinal shear steel, and similarly on the compressive side of the beam element the areas of flange relative to web also cause significant quantities of longitudinal shear steel.

The commentary to Austroads (1992)[2] Section 5 makes it clear that the design shear values used in the formula are deriving longitudinal forces related to moment gradients rather than actual levels of moment. The implication is that all material in the flange areas is necessary for the absolute level of moment present for any particular level of moment gradient. So to fully determine the necessity for longitudinal shear reinforcement perhaps one should really examine associated shears and moments. If a large shear value was associated with a relatively small moment which could be carried by the steel or concrete in the web alone (ie discounting the contribution to moment capacity of concentric prestress or flange area concrete), the implication is that no flange longitudinal shear steel may be needed. Conversely at maximum moment although there may be enough shear present (though not maximum) to require longitudinal shear reinforcement across the web flange interface, there will be much less steel required if this is less than the maximum shear. However the formulae in the Code, rightly or wrongly, are unequivocal on this point and for the Port Bouvard Bridge, maximum shear was associated with all flange material which implies maximum moment, as is directly read from the Code.

The quantity of steel required is based on factors derived from FIP (1982)[4] tests, but the mechanism is very similar to the ACI (1992)[5] shear friction formula. A comparison of these two sets of formulae shows that they give similar amounts of steel. On the face of it the steel requirements are justified, but there is one other

piece of indirect evidence which implies they may be excessive. Austroads (1992)[2] is different from AS3600 (1988)[6] in that the latter requires longitudinal shear steel checks for the webs as well as flanges, while the former requires this only for composite construction. This is because it became apparent that the steel requirement for vertical shear in the webs is almost always swamped by the longitudinal shear requirement in the web. A quick parametric study (which the authors have performed) of the vertical and longitudinal shear formulae will demonstrate the truth of this statement. The vertical shear design procedure has been established after exhaustive debate and myriads of tests by many researchers, and all of this is in effect being overthrown by one set of test results. If the web longitudinal shear steel formula is suspect, then surely the steel calculated for the flange is also suspect. In addition there must be many bridges built prior to the appearance of these formulae which do not contain this steel. Perhaps Austroads could undertake some research in this area.

5.5 Spalling Reinforcement

Another problem which the use of relatively thin webs produces is fitting in spalling reinforcement between concentric prestress anchorages in the upper and lower flanges. This would also be a problem on any launched box girder bridge as well. The problem was overcome by using pre-welded cages of reinforcement, with anchorage established by an arrangement based on tests and published by PCI[7]. This not only solved the congestion problem, but also proved to be simple to construct.

5.6 Design Aspects of the Pier Columns

An immediate difficulty encountered in the design of the pier columns was that the Code is written for the design of columns of constant cross-section only. No guidance is given for tapered column members. The particular problem this creates is the determination of the "plastic buckling load" (see Article 5.10.4.4 of Austroads (1992)[2]) which is used to evaluate the moment magnifier.

It was decided to determine the ratio of the Euler buckling capacity of the tapered column and a constant column of the same base diameter and height. This would then be multiplied by the "plastic buckling load" of the constant column to indicate a value for the tapered column. To determine the Euler capacity of the tapered column references were sought to provide standard solutions, but no cases could be found which closely resembled the particular problem. The methodology generally used for columns of varying cross-sections is to assume modes of buckling and try various solutions to find the lowest value. In some of the examples observed, quite different answers resulted from changes to the assumptions about buckling modes. The mathematics was difficult and as a first principles approach, it was not encouraging.

A more general and direct approach was adopted. A spread sheet which modelled the flexural and axial stiffness of the column was written. The level of load which gave neutral equilibrium was determined by simply observing the load at which under iterations to nth order analysis the column top transverse deflection began to diverge rather than converge. The spread sheet program gave very close agreement with the Euler load for a column of constant cross-section, but demonstrated for at least one

standard case with a varying cross-section that the buckling mode approach does not give very accurate results. Because the Euler buckling load is proportional to the constant column stiffness, the global tapered column stiffness was checked for proportionality to the Euler load for the tapered column as calculated by the spread sheet. It was found to be proportional and this conclusion gives a very simple method for determination of the Euler load of any shaped column.

6. CONCLUSIONS:

From the information presented in this paper the following conclusions may be drawn:

a. Incremental launching is a construction method which will often be favoured by contractors if given the choice in a location where it can be implemented.

b. Incremental launching tends to produce a high quality durable structure.

c. For the Port Bouvard Bridge, a large open section using a double I beam had many advantages, both for construction and for the bridge in service. This conclusion must be qualified by pointing out that the bridge width and depth was suited to such a cross-sectional arrangement in this instance, and even more so because of the requirement for lower level footways. It will not always be an ideal solution and each project must be considered in the light of its own constraints.

d. There are many advantages both in construction and for the bridge in service obtained by omitting full height web restraining diaphragms, but a penalty will be paid in transverse flexural steel, both in the webs and flanges. There will also be increasingly complex calculations.

e. The Austroads Bridge Code has two requirements which it is considered had an unduly conservative influence on the design of this structure, and similar effects are likely to occur in the design of many other structures. These are, (i) the excessive magnitude of the HLP 400 (and HLP 320) heavy load design vehicles affected many components of the structure, especially the positive longitudinal moment envelope, and (ii) the quantity of longitudinal (interface) shear steel required seems excessive. In virtually all other respects the Austroads Bridge Code is a powerful and useful document, which is at the forefront of modern world practice.

f. One other area not directly covered by the Austroads Bridge Code is the design of tall slender tapered columns. An approach was outlined which can relate this type of column to the Austroads Bridge Code ultimate limit state design methods for columns of uniform cross-section.

g. A methodology was presented for design for vessel impact using the pilecap to absorb the forces without a fendering system. This was achieved simply by shaping the pile caps. The end result was an effective, low maintenance architecturally attractive system, which added nothing to overall costs.

7. ACKNOWLEDGMENTS:

The authors would like to sincerely acknowledge the efforts of Thiess Contractors who played a significant role in the concept and design process, and who so successfully turned it into a constructed reality. They would also like to thank the Department of Marine and Harbours who were the overall Project Managers, and Main Roads Western Australia, not only for their willing cooperation and open mindedness, but also for their thorough and professional proof check. The latter task was the final design stage in this fast track project, and many times they performed well under considerable pressure.

8. REFERENCES:

1. US DoTFHA (1990). "Guide Specification and Commentary for Vessel Collision Design of Highway Bridges, Vol 1: Final Report, Vol 2: Executive Summary" US Department of Transportation Federal Highways Administration.

2. Austroads (1992). "Austroads Bridge Design Code". Austroads Publication No. AP-15.5.

3. Wyche, P.J. (1992). "Heavy Load Vehicles - Are the Austroads Bridge Code Vehicles Appropriate?" Proc 16th Australian Road Research Board Conf, Part 3, pp 19-29.

4. FIP (1982). "Guide to Good Practice - Shear at the Interface of Precast and In situ Concrete" Federation Internationale de la Precontrainte.

5. ACI (1989). "Building Code Requirements for Reinforced Concrete (ACI 318-89)" American Concrete Institute.

6. AS3600 (1988). "Concrete Structures" Standards Association of Australia.

7. PCI (1988). "Design and Typical Details of Connections for Precast and Prestressed Concrete, 2nd Ed." Prestressed Concrete Institute, Chicago.